13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3461

COMPARATIVE EXPERIMENTAL INVESTIGATION ON A R/C STRUCTURE WITH/WITHOUT DAMPED BRACES

Stefano SORACE[1] and Gloria TERENZI[2]

SUMMARY

This paper presents the results of an experimental campaign carried out on a full-scale reinforced concrete frame structure, before and after a seismic retrofit by a damped bracing system incorporating silicone fluid viscous dissipaters. This experimental enquiry was part of a European Commission-funded Research Program, named "DISPASS", developed by the pseudodynamic test method implemented at the ELSA laboratory of the Joint Research Center – Ispra. The general characteristics of the test structure, the dissipaters and their assemblage within the protective system are initially described. A special procedure adopted for the design of damped braces, as well as its application and experimental check to this case study are then presented. The main test results are finally reported, together with a performance-based evaluation of the observed response in unprotected and protected conditions.

INTRODUCTION

Incorporation of damped bracing systems into a load-bearing structural skeleton represents a viable alternative to traditional seismic retrofit techniques for reinforced concrete (R/C) buildings. Fluid viscous (FV) devices are particularly suited for this type of applications, thanks to the high level of damping action they are capable to produce in comparison to their specific volume. This is especially true for highly nonlinear devices, characterized by the lowest values (from to 0.1 to 0.2) of α exponent governing the fractional power law, which expresses the damping reaction force as a function of the relative velocity between device ends.

Several experimental studies have been developed on structures equipped by FV-damped braces, all of which consisting essentially in shaking table test campaigns on small-scale models. A comprehensive experimental program was conducted by Constantinou and Symans [1] on a 1:4 scale three-story one-bay steel frame, retrofitted by moderately nonlinear orificed fluid dampers (identified by α coefficient values greater than 0.5). This type of dampers was also used in another extensive experimental study (Reinhorn et al. [2]), where devices were incorporated in diagonal braces added to the central bay of a 1:3 scale three-story lightly reinforced concrete frame. A second shaking table campaign was subsequently carried out by Peckan et al. [3] on the same benchmark R/C model, equipped in this case with highly nonlinear silicone-FV dampers. Damped braces have always ensured remarkable reduction in interstory drifts of

1 Associate Professor, University of Udine, Udine, Italy. E-mail: stefano.sorace@dic.uniud.it
1 Researcher, University of Florence, Florence, Italy. E-mail: terenzi@dicea.unifi.it

tested models and, depending on the specific features of each experimental program, from moderate to significant reduction in base and story shears, as compared to bare frame conditions. Lately, two shaking table programs have been performed on a half-scale single-story frame incorporating first a toggle-brace-damper system (Constantinou et al. [4]), and secondly a scissor-jack-damper system (Şigaher and Constantinou [5]), constituting the first experimental verifications of these two novel damped bracing protection strategies.

This paper illustrates a pseudodynamic testing campaign carried out on a R/C frame structure in its original configuration, and then retrofitted by a damped bracing system incorporating highly nonlinear silicone-FV dissipaters manufactured by Jarret SA – France. The full-scale size of the test frame is the distinguishing feature of this experimental campaign, developed at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Center of the European Commission, as part of a European Community-funded Research Program named "DISPASS" (Dissipation and Isolation PAssive Systems Study), coordinated by the first author of this paper. After a description of the characteristics of the test structure and the protective system, as well as of the energy-based criterion adopted for this rehabilitation design and its experimental check through the experiments, a synthesis of test results is offered. A comparative evaluation of the seismic performance of the model structure with and without damped braces is finally proposed, so as to quantify the improvement of response capacities produced by the accomplished retrofit intervention.

TEST STRUCTURE AND PROTECTIVE SYSTEM CHARACTERISTICS

The test structure is a double R/C frame with three stories 2.82 m (first story) and 2.94 m (second and third) high, and two spans 6 m and 4 m wide (Sorace et al. [6], Molina et al. [7]). The two constituting frames are placed at a distance of 4.56 m. The story slabs are 240 mm thick and made of R/C joists and clay lug bricks. The primary beams are 1000 mm wide, and are comprised in the slab thickness. Columns have a mutual 400 mm x 400 mm section. The structure was designed according to the 1986 Italian Seismic Code for a peak ground acceleration of 0.25 g (Negro and Mola [8]), and contains details that are not apt to guarantee the same member ductility ensured by the most recent Italian Standard prescriptions (1996 and 2003). The column transverse reinforcement, characterized by hoops with 90-degree – instead of 135-degree – hooks, and a constant and wide spacing (200 mm) along the column height, is the poorest constructive configuration. The large base/height ratio of flat-slab beam sections is also a notably limiting factor on the ductility of these members, as well as of beam-to-column joints. The objective of the retrofit project was to provide a significant seismic improvement of the test structure, representative of a wide range of up-to-medium ductility R/C frame buildings designed between the 1986 and 1996 editions of Italian Seismic Code.

A photographic view of the structure in protected conditions, featured by an inverse-chevron brace layout incorporated in a single span, is shown in Fig. 1. The protective system configuration is identical to the one successfully adopted for a large-scale steel frame tested within the same DISPASS Project (Sorace and Terenzi [9]). The only difference between the two test models lies in the fact that the braces were not mounted on the third floor of the R/C structure, due to the relatively little benefits computed for this story against a 50% rise in costs for a total installation over the height, rather than in 2/3 of it. A detailed view of the accomplished damped brace-floor beam assemblage is included in Fig. 2. As illustrated therein, the two assumed single-acting (i.e., compression only) silicone-FV Jarret dampers are interfaced through a robust steel plate, which also connects the braces to the joint atop. By installing one device in opposite direction to the other, and by pre-stressing both up to the middle of their run, a symmetrical double-acting (i.e., tension-compression) dynamic response is obtained, as it is illustrated in the next section. Thorough information on the behavioral characteristics of this class of dampers, as well as on their analytical and numerical modeling, can be found in Terenzi [10], and Sorace and Terenzi [11], [12].

Fig. 1. View of test structure.

Fig. 2. View of a brace-damper-beam connection.

DESIGN CRITERION OF PROTECTIVE SYSTEM

The design criterion of FV dissipaters incorporated in damped braces – whose general lines have been defined in Sorace and Terenzi [13] – consists in assigning them the capability of dissipating a prefixed share of the total seismic input energy computed for the structure on each floor. This concept, developed within a typical nonlinear dynamic analysis approach, is summed up in the following expression:

$$E_{D\mathrm{j}} = \beta_{\mathrm{j}} E_{I\mathrm{j}} \tag{1}$$

where β_j is the energy ratio for the j-*th* floor, selected on the basis of a performance assessment analysis of the structure including the braces only, but without protective devices;

$$E_{D\mathrm{j}} = \int_{0}^{t_f} c_{\mathrm{j}} \left|\dot{v}_{\mathrm{j}}\right|^{\alpha} sgn(\dot{v}_{\mathrm{j}})\dot{v}_{\mathrm{j}} dt \tag{2}$$

is the energy dissipated on the j-*th* floor by the set of dampers placed on that floor, being: c_{j} = global damping coefficient, characterizing the j-*th* floor dampers; $\dot{v}_{\mathrm{j}}$ = relative velocity of the j-*th* floor; |•| = absolute value; $sgn(\cdot)$ = signum function; α = fractional exponent defining the damping reaction force of the j-*th* floor $F_{D\mathrm{j}}$, given by:

$$F_{D\mathrm{j}} = c_{\mathrm{j}} \left|\dot{v}_{\mathrm{j}}\right|^{\alpha} sgn(\dot{v}_{\mathrm{j}}) \tag{3}$$

and

$$E_{I\mathrm{j}} = \int_{0}^{t_f} m_{\mathrm{j}} \ddot{v}_{t\mathrm{j}} dv_g \tag{4}$$

is the "absolute" input energy (Uang and Bertero [14]) of the j-*th* floor, being: m_{j} = mass associated to the j-*th* floor; $\ddot{v}_{t\mathrm{j}}$ = absolute j-*th* floor acceleration; and v_g = ground displacement. In design analyses, the energy balance expressed by (1) is computed in mean terms over the response to the set of input ground motions adopted in the supporting nonlinear dynamic enquiry.

The process starts from an initial evaluation of the input energy at the j-*th* floor for the numerical finite element model already including the braces but without dampers $E_{I\mathrm{j}}^{wd}$, so as to consider the stiffening action of bracing, and thus its effects on the seismic response and the balance given by (1), from the beginning of the analysis. Afterwards, an iterative search analysis is conducted to locate the set of c_{j} values ensuring attainment of the β_{j} coefficient distribution established by (1). For each input ground motion, the relevant calculation is developed by a least-square recursive scheme detailed in Sorace and Terenzi [13]. The energy balance (1) can in principle be performed at any instant of the response time-history. Following the suggestions reported in Sorace and Terenzi [13], the evaluation is carried out herein for the instant at which the j-*th* floor reaches maximum interstory drift $ID_{\mathrm{j},max}$.

The practical application of this criterion implies, first of all, a proper choice of the set of βj values governing (1). This can be calibrated on the $ID_{\mathrm{j},max}$ reduction required in the design phase when passing from unprotected (denoted by superscript "up") to protected ("p") conditions, as expressed by the performance ratio $r_{ID\mathrm{j}} = ID_{\mathrm{j},max}^{up} / ID_{\mathrm{j},max}^{p}$. By referring to the floor subjected to the highest drift demand in the original structure, defining $\beta_{\max}$ as the relevant energy ratio coefficient, the following tentative values can be adopted for the r_{ID} ranges of technical interest, in the case of R/C frame buildings: $\beta_{\max} = 0.5$ for $r_{ID} = 1.5 \div 1.8$; $\beta_{\max} = 0.6$ for $r_{ID} = 1.8 \div 2.2$; and $\beta_{\max} = 0.7$ for $r_{ID} = 2.2 \div 2.5$. These $\beta_{\max}$ choices should be increased by around 15%, for the same r_{ID} ranges, when damped braces are not mounted on the upper floor(s). For low-to-medium rise structures, once $\beta_{\max}$ has been selected, β_{j} values for the remaining floors can be fixed by subdividing the building in three zones along its height. Afterwards, by taking into account the typical energy and interstory drift profiles that characterize the response of rather regular frame structures, $\beta_{\max}$ can be extended to all the stories of the central zone – where the floor subjected to the highest drift demand generally belongs –, whereas $\beta_{\mathrm{j}} = 0.7 \div 0.8\ \beta_{\max}$ can be chosen for the lower zone, and $\beta_{\mathrm{j}} = 0.3 \div 0.4\ \beta_{\max}$ for the upper one. When the protective system is not installed on the upper zone, β_{j}

= β_{max} can be selected for the lower one. In the case of irregular or tall buildings, a more articulated partition could be hypothesized along the height, although based on the same principles.

APPLICATION OF DESIGN CRITERION TO TEST STRUCTURE RETROFIT

A preliminary inelastic seismic assessment analysis developed on the bare structure allowed locating the maximum interstory drift demand on the second floor. The computed drift value under the input accelerogram assumed in the experimental investigation – scaled at the basic design level of 0.25 g – was equal to 45 mm, corresponding to a drift ratio (i.e., the ratio of interstory drift to interstory height) of 1.53%. The fundamental design assumption consisted in reducing this value below 1%, nearly coinciding with the elastic limit threshold, which led to look for a r_{ID_2} reduction factor in the order of 1.6÷1.8. Consistently with the suggestions reported in the previous section, and considering that the upper floor was not braced, a target energy-share coefficient of 0.6 was fixed for the second floor, and extended to the first one ($\beta_1 = \beta_2 = \beta_{max} = 0.6$). The corresponding damping coefficient demands evaluated through the iterative search process associated to the design criterion were: $c_2 = 5.87$ kN·(s/mm)$^{\alpha}$, $c_1 = 5.76$ kN·(s/mm)$^{\alpha}$, being $\alpha = 0.2$. BC1FN-type Jarret dampers were then adopted, characterized by a damping coefficient of 6 kN·(s/mm)$^{\alpha}$, which is closer to the computed c_1 and $c_2$values.

TEST PROGRAM, EXPERIMENTAL CHECK OF DESIGN CRITERION AND NUMERICAL MODEL CALIBRATION

The experimental program was articulated on the following four pseudodynamic tests: (1) in unprotected conditions, with the input accelerogram scaled at the original amplitude of 0.25 g, corresponding to the basic-design earthquake – with a 10% probability of being exceeded over 50 years – for this retrofit project (denoted as BSE-1 earthquake, according to FEMA 273 guidelines for the seismic rehabilitation of building structures [15]); (2) in protected conditions, with BSE-1 earthquake as input; (3) as (2), but with the accelerogram scaled at 80% of the original amplitude, that is, at 0.2 g, to simulate an intermediate event with a 20% probability of exceedance over 50 years (named IE-1); (4) as (2), but with the accelerogram scaled at 120%, that is, at 0.3 g, to simulate an intermediate event with a 5% probability of exceedance over 50 years (named IE-2).

The experimental campaign was developed by the pseudodynamic test method implemented at the ELSA laboratory, characterized by an "accelerated continuous" on-line architecture (Magonette et al. [16]). This basic feature, combined with a special damper restoring-force compensation technique (Molina et al. [7]), allowed virtually annulling the strain rate effects that typically affect the response of rate-sensitive devices, such as the Jarret ones, when subjected to quasi-static or "classic" pseudodynamic tests, in place of real-time dynamic loading processes. More complete information about the experimental program, as well as on the above-mentioned compensation technique and relevant characterization tests executed on the dampers prior to the installation onto the structure, is reported in Sorace et al. [6] and Molina et al. [7].

The results of the application of the design criterion are demonstratively illustrated, for the second floor, in Fig. 3. Therein, the numerically computed energy time-histories are plotted for the undamped braced structure $E_{I2,num}^{wd}$, and the energy dissipated by the dampers placed on the same floor in protected conditions $E_{D2,num}$, evaluated by introducing in the numerical model the exact damping factors of the actually installed dissipaters. Furthermore, the experimentally dissipated energy $E_{D2,exp}$, as derived from test results, is traced out for comparison. The $E_{I2,num}^{wd}$ and $E_{D2,num}$ values for the instant at which the maximum interstory drift $ID_{2,max}$ is reached, are also located in these curves. The first observation prompted by the energy values obtained is that the numerically calculated β_2 coefficient, expressed as:

$\beta^{c}_{2,num} = E_{D2,num}(ID_{2,max}) / E^{wd}_{I2,num}(ID_{2,max})$, is equal to 0.62. This value matches the target β_2 coefficient imposed in the design criterion. This holds true also for the first story energy ratio: $\beta^{c}_{1,num}$ = 0.64, very close to the corresponding target β_1 value.

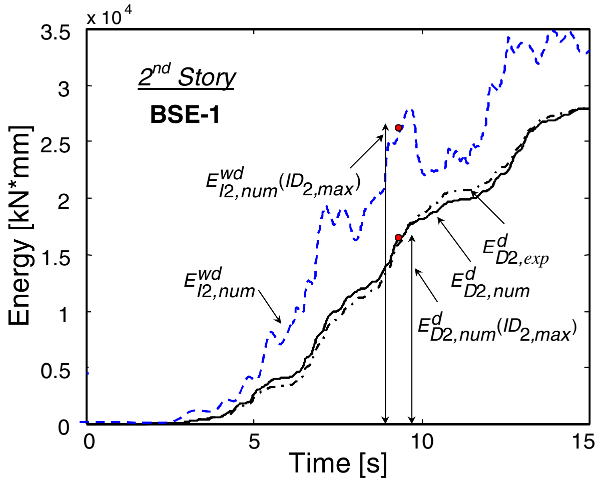

Fig. 3. Second story energy time-histories and location of values corresponding to the instant of maximum interstory drift.

A second aspect, directly highlighted by the graphs in Fig. 3, concerns the experimentally dissipated energy at the instant of maximum interstory drift, which coincides with the numerical one. Indeed, also the "experimentally evaluated" β_2 coefficient, $\beta^{c}_{2,exp} = E_{D2,exp}(ID_{2,max}) / E^{wd}_{I2,num}(ID_{2,max})$, is equal to 0.62. At the same time, $\beta^{c}_{1,exp}$ = 0.65 resulted for the first floor. This total correlation between experimentally and numerically evaluated energy-share coefficients is the consequence of the effective reproduction of the structure and Jarret damper response ensured by the adopted numerical finite element model (Sorace and Terenzi [11]). This correlation is displayed in Figs. 4 and 5, showing a direct comparison between numerical and experimental second floor interstory drift time histories (Fig. 4), and response cycles of the set of dampers installed on the same floor (Fig. 5), as way of example, for the BSE-1 test in retrofitted conditions. The graphs in Fig. 5 also highlight the typical rounded parallelogram-like cycles produced by each couple of interfaced Jarret dampers, as well as their complete stability at the different excitation levels imposed by the input accelerogram. By introducing some of the test results that will be discussed in the next section, the following maximum interstory drifts were obtained under BSE-1 earthquake: 43.1 mm (unprotected), and 25.2 mm (protected), for the second floor; 30.7 mm (unprotected), and 17 mm (protected), for the first one. This determines the following performance ratios: r_{ID_2} = 1.71, and r_{ID_1} = 1.8, both belonging to the searched 1.6÷1.8 range. Limitedly to the present design application, these data corroborate the suggested β_{max} choices for the r_{ID} range of interest.

PERFORMANCE EVALUATION OF EXPERIMENTAL RESPONSE WITH/WITHOUT DAMPED BRACES

The response obtained from the four tests is summed up, in terms of $ID_{j,max}$ and maximum interstory shears $V_{j,max}$, in Table 1.

The following observations can be drawn from these data and their simple elaborations:

- In addition to the second and first interstory drift reductions quantified by r_{ID_2} and r_{ID_1} ratios reported in the previous section – with the former causing a totally elastic, instead of decidedly plastic second floor response –, non negligible benefits also derive for the unprotected third floor, for which a r_{ID_3} value of 1.13 is obtained by comparing BSE-1 tests before and after retrofit,
- As a consequence, the third floor drift ratio, equal to 0.94%, is kept within the same 1% threshold assumed as a design objective for the second floor;
- This threshold is overcome by around 20% only in the IE-2 test at 120% ground motion amplitude, giving rise to a slightly plastic activity of the third floor in this case; at the same time, the first and second floor response remains totally elastic, also for this maximum input amplitude;
- The peak third story drift recorded in the IE-2 test is also 19% lower than the second story peak drift for 100% BSE-1 test in unprotected conditions;
- The ratio of $ID_{2,max}$ to $ID_{3,max}$ is equal to 1.42 for the original structure, whereas the ratio of $ID_{3,max}$ to $ID_{2,max}$ does not exceed 1.21 in the three tests in retrofitted configuration; this produces a more regular pattern of story drift demand along the height, although in the absence of braces at the third floor.

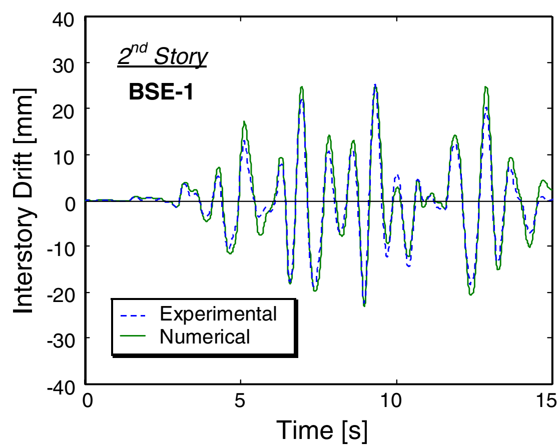

Fig. 4. Comparison of experimental and numerical second story interstory drift time-histories for BSE-1 test in retrofitted conditions.

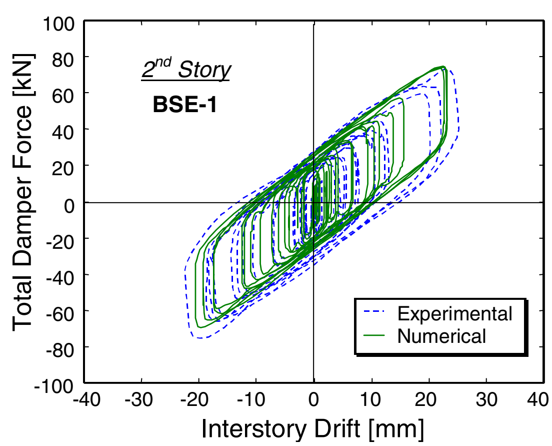

Fig. 5. Comparison of experimental and numerical second story damper hysteresis cycles for BSE-1 test in retrofitted conditions.

Table 1. Maximum experimental interstory drifts and shears.

Test No.	Story	$ID_{j,max}$ (%)	$V_{j,max}$ (kN)
BSE-1	1	30.7	496
Unprotected	2	43.1	434
	3	31.0	300
BSE-1	1	17.0	466
Protected	2	25.2	390
	3	27.5	309
IE-1	1	11.8	325
Protected	2	16.7	256
	3	21.6	213
IE-2	1	22.9	651
Protected	2	29.2	634
	3	35.1	381

A visual representation of the benefits induced by the retrofit interventions in terms of interstory drifts and second floor hysteresis cycles is shown in Figs. 7 and 8, where the experimental responses to BSE-1 are comparatively plotted for the original structure and the protected configuration. In addition to the observations above, the seismic improvement produced by the incorporation of damped braces was also assessed by a formal performance-based evaluation analysis. This was carried out by referring to the combination of five structural (S-1 – Immediate Occupancy; S-2 – Damage Control; S-3 – Life Safety; S-4 – Limited Safety; S-5 – Collapse Prevention) and four non-structural (N-A – Operational; N-B – Immediate Occupancy; N-C – Life Safety; N-D Hazards Reduced) performance levels and ranges proposed in FEMA 273. This combination gives rise to twenty building performance levels (from 1-A, as resulting from the combination of S-1 with N-A, to 5-D, from S-5 and N-D). The interstory drift was adopted as the reference response parameter for this evaluation enquiry. Based on a critical review of suggestions provided for R/C frame structures in FEMA 273, as well as in other prominent design codes and guidelines for new construction and seismic rehabilitation, the following interstory drift ratios were selected as the corresponding limitations: 1% − S-1; 1.5% − S-2; 2% − S-3; 3% − S-4; >3% − S-5, for structural levels; and 0.3% − N-A; 0.5% − N-B; 1% − N-C; 2% − N-D; 3% − N-E, for non-structural levels. Crossing these two sets of values, the following limits for building performance levels were then obtained, up to a drift ratio of 3%: 0.3% – 1-A (1% – S-1$\cap$ 0.3% – N-A); 0.5% – 1-B; 1% – 1-C; 1.5% – 2-D; 2% – 3-D; 3% – 4-E.

Experimental response data were integrated with an additional numerical enquiry carried out by the finite element model calibrated against test results. Six analyses were developed: in protected and unprotected conditions, with the input accelerogram scaled at two further input amplitudes, equal to 50% (serviceability earthquake – SE) and 150% (maximum considered earthquake, or second basic design earthquake – BSE-2) of the original one, characterized by a 50% and 2% probability of being exceeded over 50 years, respectively; and in unprotected configuration, with IE-1 and IE-2. A total of ten (four experimental plus six numerical) analyses were then obtained, whose representative points in terms of interstory drift ratio are plotted for the second and third floor in Fig. 8, as a function of the input motion level. The response points are compared in this graph with the building performance drift limits defined above, giving rise to the formal performance evaluation summed up in Table 2. The curves in Fig. 8 show a nearly stable improvement of seismic performance over the five hazard levels, with an average 1.68 reduction factor between the second floor drifts in original conditions and the third floor drifts in the presence of a protective system. This is reflected in Table 2 by the methodical one-level shift of building

performance, and thus by the attainment of an enhanced-performance multiple design objective, after retrofit.
This objective is consistent with the rehabilitation intervention purposes, which validates the assumption of limiting the installation of the protective system on the two lowest stories only, in this case.

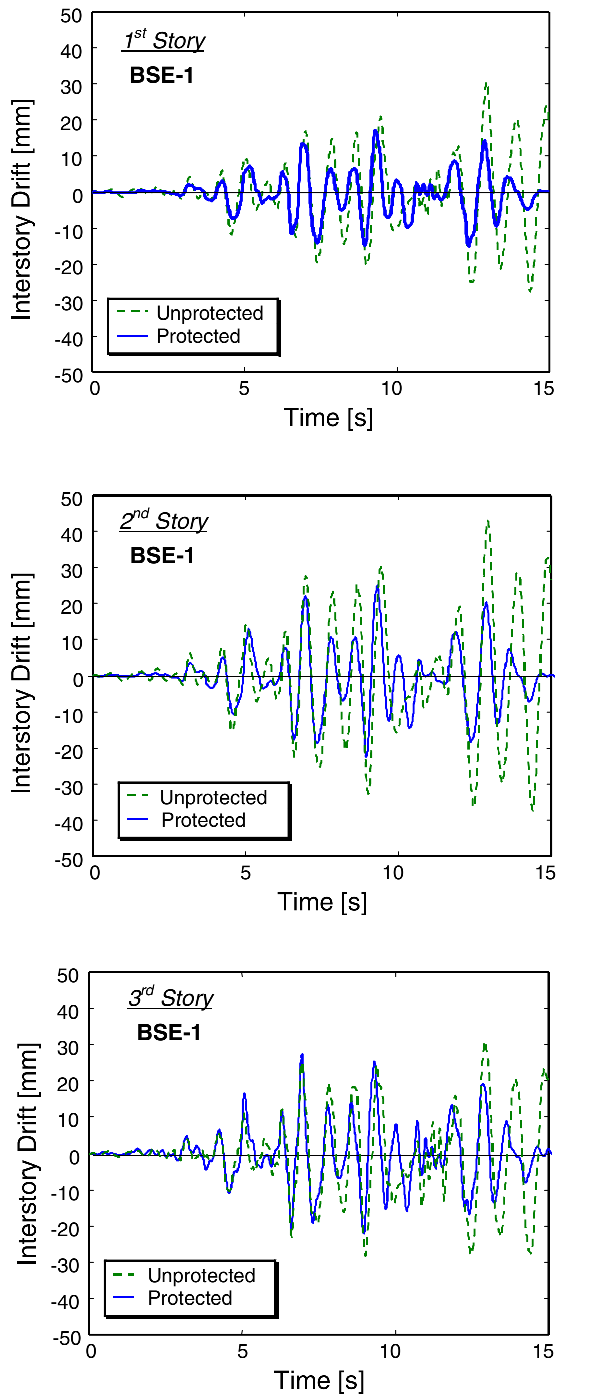

Fig. 6. Comparison of experimental interstory drift time histories obtained from BSE-1 tests in unprotected and protected conditions.

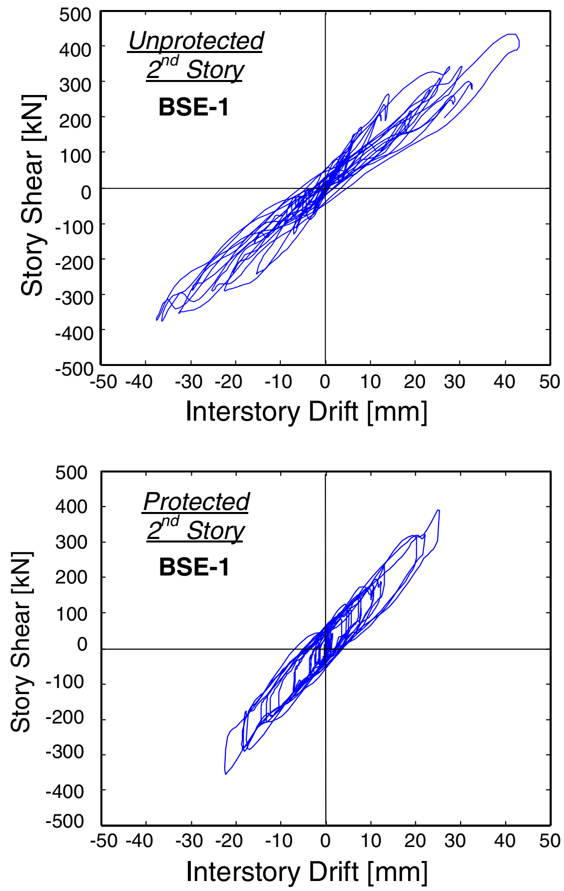

Fig. 7. Experimental second story hysteresis cycles obtained from BSE-1 tests in unprotected and protected conditions.

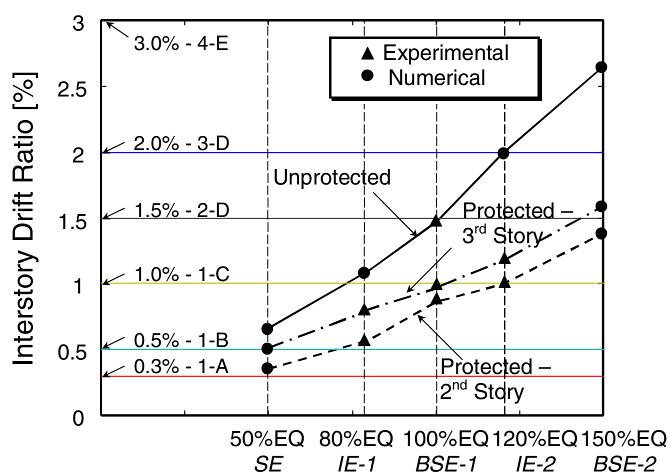

Fig. 8. Interstory drift ratio curves as a function of input ground motion level, and comparison of response points with assumed performance limitations.

Table 2. Building performance levels evaluated for the second story in unprotected conditions and the third story in protected conditions.

Hazard Level	Building Performance Level	
	Unprotected 2nd Story	Protected 3rd Story
SE	1-C	1-B
IE-1	2-D	1-C
BSE-1	2-D	1-C
IE-2	3-D	2-D
BSE-2	4-E	3-D

CONCLUDING REMARKS

The pseudodynamic experimental campaign carried out on the R/C frame structure tested at the ELSA laboratory as part of the "DISPASS" Program allowed reaching a series of objectives formulated for this section of the entire research project, which can be recapitulated as follows:

– An effective application of the energy-based design method formulated for FV-damped braces, and a realistic system installation and verification of its operation under severe seismic inputs, were obtained thanks to the full-scale dimensions of the test structure;
– A satisfactory experimental check of the proposed application criteria of the design method, as well as of its prediction potentialities, was derived for this case study, as highlighted by the strict correlations between target, post-calculated and experimentally computed values of the governing energy-share coefficients;
– A direct comparison of the response capacities in original and protected conditions was developed for the two tests under the basic design earthquake, showing a significant improvement in terms of interstory drifts after retrofit;
– Two additional tests were conducted in protected configuration, which allowed fixing two further experimental points on the interstory drift ratio-hazard level curves drawn in the performance evaluation section of this study;
– These curves were completed by the results of the numerical analyses carried out with the structural model calibrated against test results, showing nearly constant benefits in passing from original to rehabilitated conditions for all the five considered hazard levels;
– The achieved improvements of seismic performance well matched the design objectives of the retrofit intervention;
– This also validated the hypothesis of mounting the protective system on the two lowest stories only, with a considerable reduction of costs in comparison with a total installation over the structure height.

REFERENCES

1. Constantinou MC, Symans MD. "Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers". Report No. NCEER-92-0032, National Center for Earthquake Engineering Research, Buffalo, NY, 1992.
2. Reinhorn AM, Li C, Constantinou MC. "Experimental and analytical investigation of seismic retrofit of structures with supplemental damping: Part 1–Fluid viscous damping devices". Report No. NCEER-95-0001, National Center for Earthquake Engineering Research, Buffalo, NY, 1995.

3. Peckan G, Mander JB, Chen SS. “The seismic response of a 1:3 scale model R.C. structure with elastomeric spring dampers”. Earthquake Spectra 1995; 11(2): 249-267.
4. Constantinou MC, Tsopelas P, Hammel W, Sigaher AN. “Toggle-brace-damper seismic energy dissipation systems”. Journal of Structural Engineering, ASCE 2001; 127(2): 105-112.
5. Şigaher AN, Constantinou MC. “Scissor-Jack-Damper energy dissipation system”. Earthquake Spectra 2003; 19(1): 133-158.
6. Sorace S, Introini G, Magonette G, Molina FJ, Terenzi G. “Analisi sperimentale di strutture a telaio dotate di controventi dissipativi”. Proceedings of the 14th Italian Congress on Building Technology – CTE, Mantua, Italy. Milan: CTE, 2002: 401-410 (in Italian).
7. Molina FJ, Magonette G, Viaccoz B, Sorace S, Terenzi G. “Pseudo-dynamic tests on two buildings retrofitted with damped braces”. Publication EUR 20993 EN, European Commission – Joint Research Centre, Institute for the Protection and Security of the Citizen/European Laboratory for Structural Assessment, Ispra, Italy, 2003.
8. Negro P, Mola E. “Current assessment procedures: application to regular and irregular structures compared to experimental results”. Proceedings of the 3rd European Workshop on the Seismic Behaviour of Irregular and Complex Structures. Florence: EAEE, 2002.
9. Sorace S, Terenzi G. “Large-scale experimental validation of a design procedure for damped braced steel structures”. Proceedings of STESSA 2003 – 4th International conference on the Behaviour of steel structures in seismic areas, Naples, Italy. Lisse: AA Balkema 2003: 657-662.
10. Terenzi G. “Dynamics of SDOF systems with nonlinear viscous damping”. Journal of Engineering Mechanics, ASCE 1999; 125(8): 956-963.
11. Sorace S, Terenzi G. “Non-linear dynamic modelling and design procedure of FV spring-dampers for base isolation”. Engineering Structures 2001; 23(12): 1556-1567.
12. Sorace S, Terenzi G. “Non-linear dynamic design procedure of FV spring-dampers for base isolation – Frame building applications”. Engineering Structures 2001; 23(12): 1568-1576.
13. Sorace S, Terenzi G. “Iterative design procedure of fluid viscous devices included in braced frames”. Proceedings of EURODYN ’99 – 4th European Conference on Structural Dynamics, Prague Czech Republic. Rotterdam: AA Balkema, 1999: 169-174.
14. Uang CM, Bertero VV. “Use of energy as a design criterion in earthquake-resistant design”. Report No. UCB- EERC 88/18. University of California at Berkeley, Berkeley, CA, 1988.
15. NEHRP “Guidelines for the seismic rehabilitation of buildings”. FEMA Publication No. 273, Federal Emergency Management Agency, Washington DC,1997.
16. Magonette G, Pegon P, Molina FJ, Buchet Ph. “Development of fast continuous substructuring tests”. Proceedings of the 2nd World Conference on Structural Control, Tokyo, Japan, 1998.

ACKNOWLEDGEMENTS

The activities reported in this paper are part of the Research Program “DISPASS”, funded by the European Community within the framework of the ECOLEADER Consortium – Contract No. HPR-CT-1999-00059. The authors also acknowledge Mr Gianni Introini for having supplied the devices used in the experimental campaign.